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FINAL
QUINAULT ESTATES DRAINAGE INVESTIGATION

Volume I - Text

December 1993

R.W. BECK
AND ASSOCIATES

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QUINAULT ESTATES
DRAINAGE INVESTIGATION

FINAL REPORT

PREPARED FOR:

KING COUNTY DEPARTMENT OF PUBLIC WORKS
SURFACE WATER MANAGEMENT DIVISION

PREPARED BY:

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DECEMBER 1993

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R. W. Beck and Associates

APPROVED BY: Bruce Johnson
King County

QUINAULT ESTATES DRAINAGE INVESTIGATION

CERTIFICATE OF ENGINEER

The engineering material and data contained in this study were prepared under the supervision and direction of the undersigned, whose seals as Registered Professional Engineers are affixed below.

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QUINAULT ESTATES INVESTIGATION

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VOLUME II - TECHNICAL APPENDICES (Bound Separately)

VOLUME III - STUDY AREA PHOTOGRAPHS (Separate Photograph Notebook)

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SECTION I

INTRODUCTION AND PURPOSE

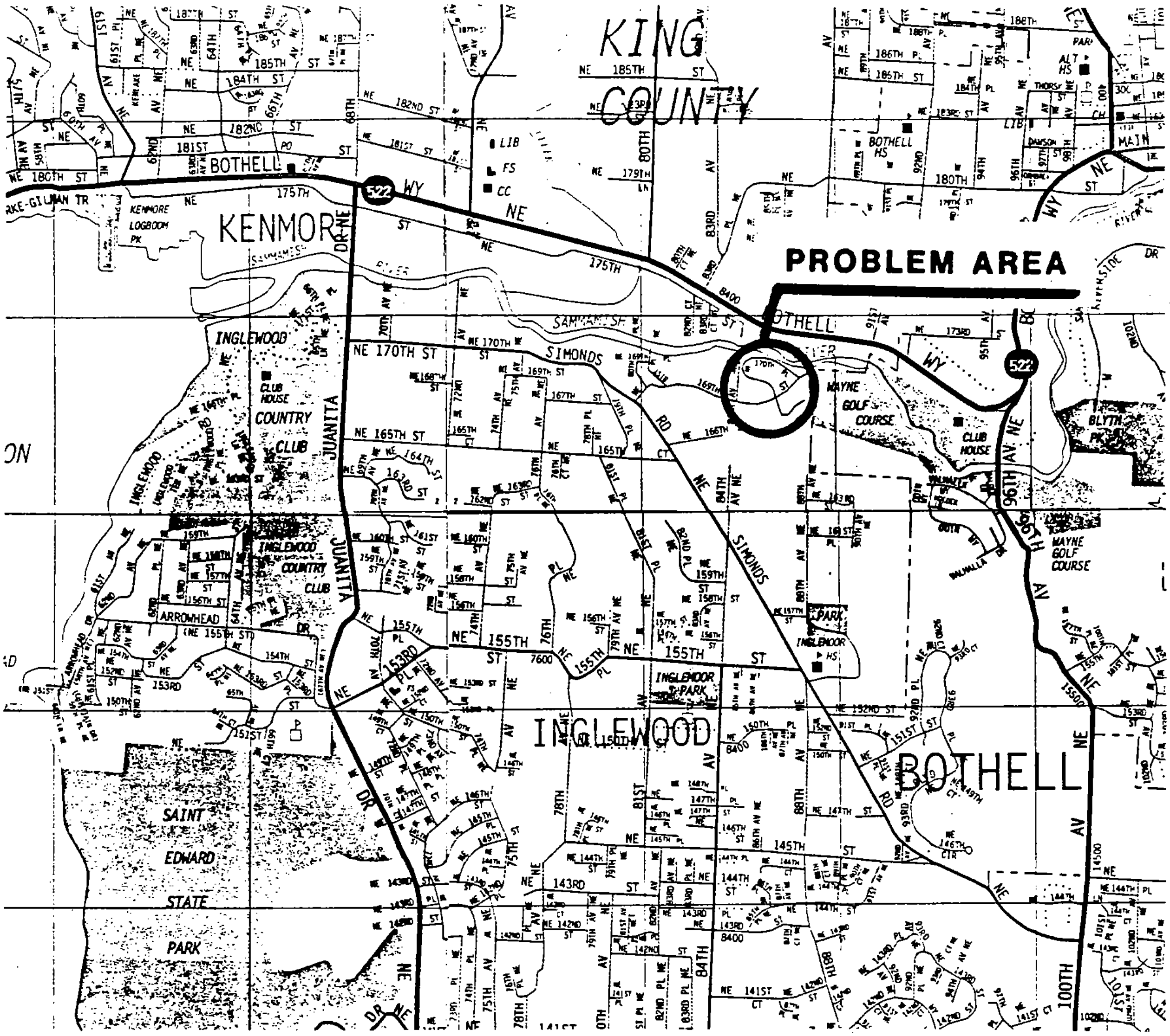
A. INTRODUCTION

This study includes an investigation of the Quinault Estates area which has a long history of drainage problems resulting in claims and lawsuits. Over the last 15 years, the County has responded with studies, several small capital improvements, and a heightened level of County maintenance and monitoring during storm events. In addition, one large capital improvement was designed but not constructed due to an inability to obtain an easement.

The problems result from three small, steep ravines that continue to erode and deposit debris and sediment in the Quinault Estates subdivision. Quinault Estates was constructed on an alluvial fan below the ravines, adjacent to the Sammamish River. The ravines are very susceptible to landslides and stream erosion. Streams in each ravine transport debris and erodible material downstream to the Quinault Estates pipe drainage systems, which convey flows and sediment to the Sammamish River. Sediment deposition and debris accumulation have resulted in occasional plugging of the pipe drainage systems, especially at the pipe system entrances. Plugging of these systems has resulted in flooding and sediment deposition on public and private property, including residences. The problem area vicinity is illustrated in Figure 1. The Quinault Estates subdivision, the ravines, referred to as the west ravine, middle ravine and east ravine, and the study area, are illustrated in Figure 2.

Although the County has constructed some improvements, all three ravines continue to be subject to erosion and sediment deposition. During the recent July 13, 1993 event, drainage systems below all three ravines overflowed. Flooding and sediment deposition problems from the middle ravine were the most severe. Approximately 20 homes and several roads received flooding and sediment deposition. The County has so far received about $200,000 in claims from this event. The July 13 event had a total precipitation of 1.67 inches, 1.43 inches of which occurred in a one-hour period, corresponding to a 100-year 1-hour precipitation. Photographs 1, 2, and 3, following page II-2, show the results of this flood. The approximate limits of historic flooding and sediment deposition are illustrated in Figure 3.

R. W. Beck and Associates was retained by SWM to conduct an investigation of the flooding and sediment deposition problems. The study includes development of alternative solutions for alleviating the problems and recommendations for preferred alternatives based on feasibility, impact to the downstream system, project costs, advantages and disadvantages, and environmental considerations.



No Scale

FIGURE 1
KING COUNTY
SURFACE WATER MANAGEMENT DIVISION
QUINAULT ESTATES STUDY

VICINITY MAP

R.W. BECK AND ASSOCIATES

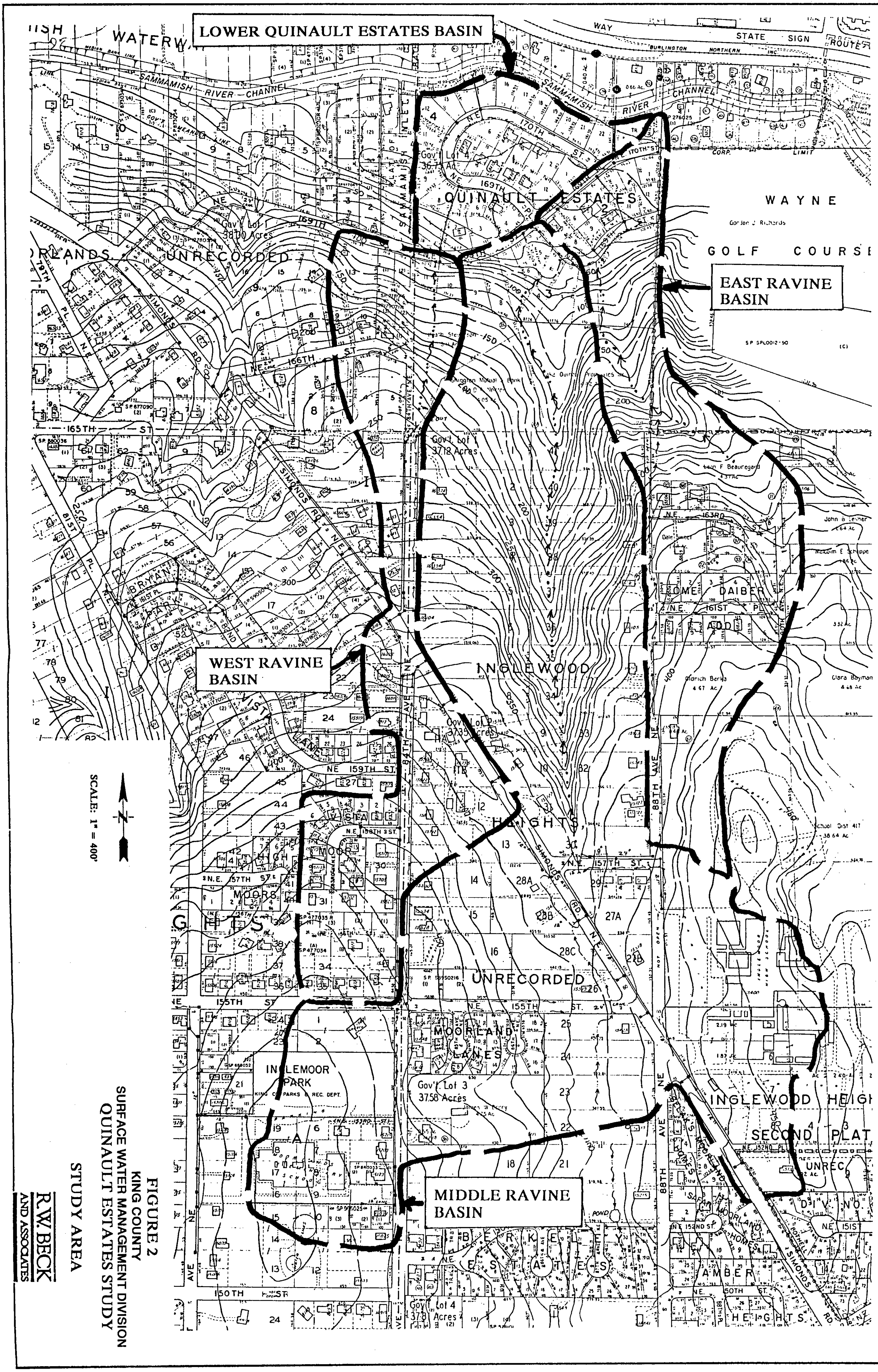
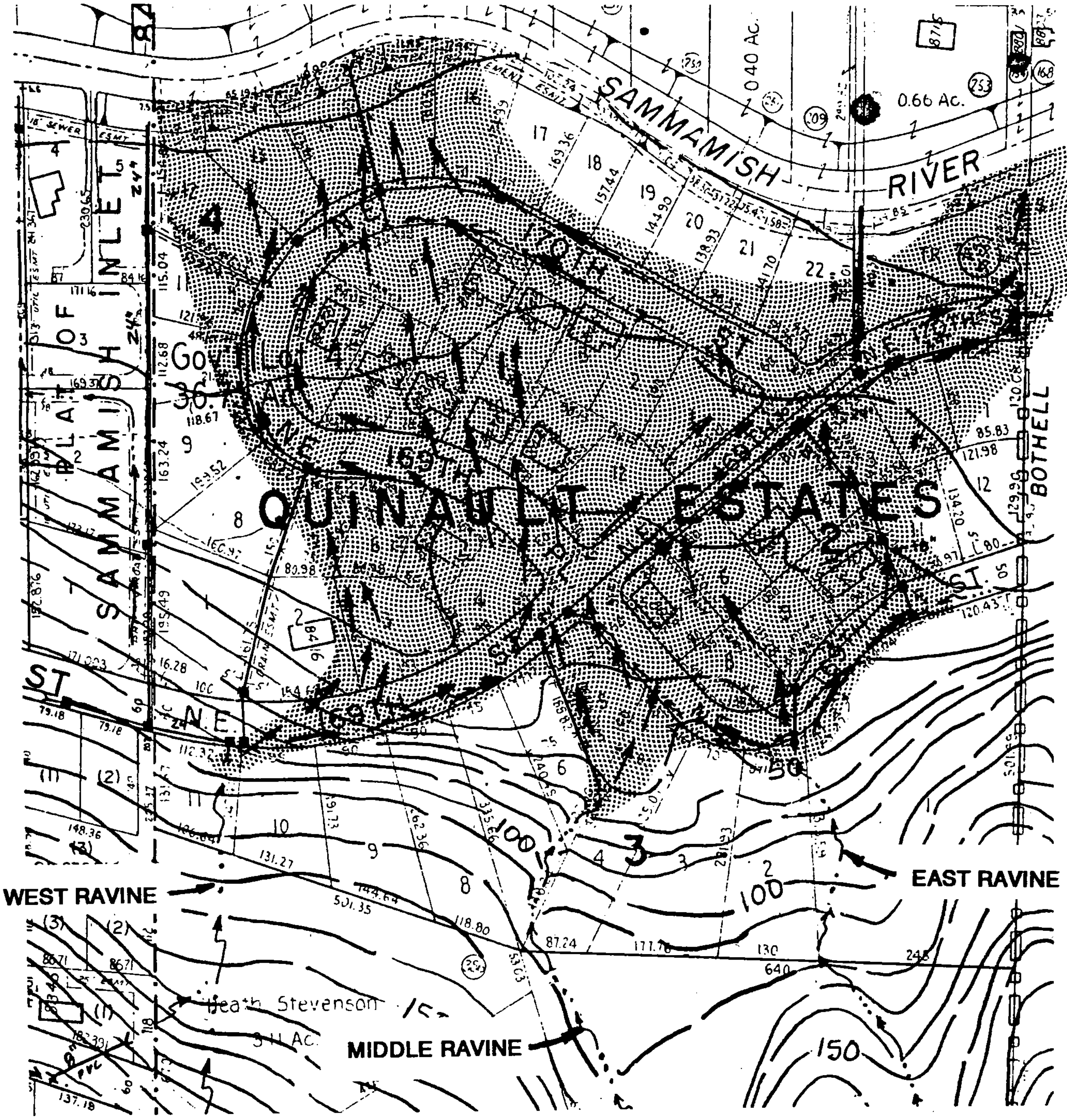


FIGURE 2
KING COUNTY
SURFACE WATER MANAGEMENT DIVISION
QUINAULT ESTATES STUDY

STUDY AREA

R.W. BECK AND ASSOCIATES



APPROXIMATE AREAS OF HISTORIC FLOODING AND/OR SEDIMENT DEPOSITION PROBLEMS. ARROWS INDICATE APPROXIMATE FLOW PATHS DURING FLOODING EVENT.

No Scale

FIGURE 3
KING COUNTY
SURFACE WATER MANAGEMENT DIVISION
QUINAULT ESTATES STUDY

HISTORIC PROBLEM AREAS

R.W. BECK AND ASSOCIATES

B. Purpose

R. W. Beck was retained by King County Surface Water Management Division (SWM) to conduct a drainage investigation of this problem. The purpose of this study was to:

- Verify the existence and severity of the problem.
- Assess the level of protection (LOP) provided by the existing drainage system.
- Identify the probable cause of the problems.
- Evaluate active geologic processes.
- Identify and evaluate alternatives for solving the problem.
- Recommend preferred alternative solutions.

C. Authorization

Preparation of this study was authorized by an engineering agreement with R. W. Beck and Associates dated July 2, 1993.

D. Acknowledgments

King County Project Manager and representative was Bruce Johnson. Input was also received from other King County SWM staff as well as the County Road Maintenance personnel.

The project team from R. W. Beck and Associates included Jack C. Bjork as Project Manager and Mike Giseburt as Project Engineer.

Assistance and input on the geological investigations were provided by Robert Palmquist at Applied Geotechnology Incorporated (AGI), and John Bethel of King County. Portions of the AGI report were used in this text.

SECTION II

BACKGROUND / HISTORY

A. Complaints / Claims / History

The Quinault Estates subdivision was platted in 1962. Most of the residences, however, were constructed after 1970. The original drainage systems have been significantly modified over the years. The 1962 drainage plans show that stream flows from the middle and east ravine originally joined together on the south side of 169th Place NE and NE 170th Street. From there, the flows traveled northeast via ditches and a half-round pipe along the south side of NE 170th Street 300 feet, then veered north and discharged to a low gradient ditch along the east side of Tract A. Tract A has been developed and is known as the Bondelid Property (Bondelid has since moved).

The west ravine flowed south to 169th Avenue NE where it entered an 18-inch-diameter pipe system which flowed through drainage easements to the Quinault Estates roadway ditch system and ultimately to the Sammamish River between lots 14 and 15 of Division 4. Refer to the large fold-out basin map (Figure 4) at the end of this report.

The half-round pipe system conveying the middle and east ravine flows was damaged in a 1974 storm and was replaced with a 24-inch diameter CMP. The new pipe was undersized and had a flat slope and soon filled with sediment, resulting in recurrent roadway flooding.

Between 1975 and 1977 the County received several complaints and allegations that the County had diverted additional runoff to the west ravine as a part of an improvement project along Simonds Road. Based on a review of the 1977 Simonds Road "as-builts" plans, it appears that 22 acres south of Simonds Road were diverted to the 84th Avenue NE drainage system and ultimately to the west ravine. Following the lawsuit, the County constructed a new 24-inch-diameter pipe system north of 169th Avenue NE to convey the west ravine flows around the Quinault Estates development along the 84th Avenue NE extension to the Sammamish River. The existing 18-inch-diameter system was abandoned and left to convey only runoff generated from within the Quinault Estates development.

About 1977, the County constructed a second project to correct problems with the 24-inch-diameter CMP pipe carrying middle and east ravine flows. The project included a new 30-inch-diameter concrete pipe system to convey middle ravine flows to the Sammamish River, leaving the 24-inch-diameter CMP pipe to convey east ravine flows. The 30-inch-diameter system discharges to the Sammamish River 220 feet west of the east ravine discharge to the river. The project also included a 96-inch-diameter manhole near the river to serve as a sediment trap. The manhole was too small and proved ineffective in removing sand and fine-grained material, and a bar soon formed in the river at the pipe outlet. Sedimentation in the stream along the east side of Tract A (carrying east ravine flows) also became a problem, and a bar formed in the river in front of this property. The County then began dredging in the Sammamish River to maintain the channel in front of the two residences.

As awareness of the negative environmental impacts of dredging grew, the Department of Fisheries (DOF) issued a final dredging permit in 1987, conditioned on the County's commitment to address the sedimentation problem in the Sammamish River with a long-term solution. In the fall of 1987, King County Roads Division built a small sedimentation trap at the base of the east ravine on the south side of NE 169th Street. An investigation by SWM - PM&D in the spring of 1988 indicated that most of the sediment was produced by the middle ravine, and alternatives to solve the problem were evaluated. The County decided to enlarge the existing sediment trap for the east ravine and to install an additional sedimentation vault in the easement downstream of a 96-inch diameter manhole for the middle ravine. The additional vault was installed in 1988. However, planned upgrades to the sediment trap at the bottom of the east ravine were halted due to problems acquiring necessary right-of-way from the owner of lot 2 Division 3.

Sedimentation in these systems is still an ongoing problem requiring maintenance at least once a year. The systems are monitored by the County during and following significant storms. To further decrease the sediment deposition in the Sammamish River from the middle ravine, SWM recently installed a second sediment vault in November. This vault was installed within the drainage easement just downstream of the other sediment vault.

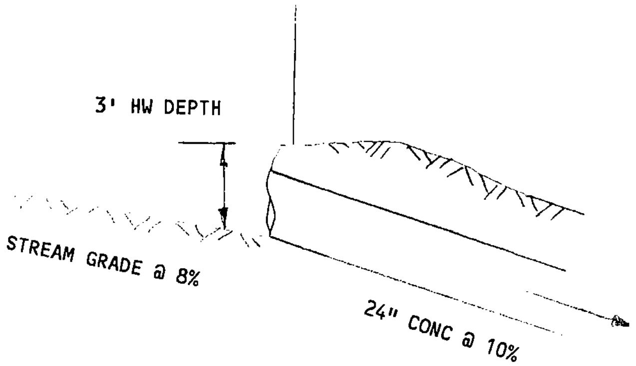
As previously mentioned, a significant storm event occurred this year on July 13, 1993. A summary of the observations made during this event is given in Section IV.

B. Basin Description

1. General

Each of the ravines convey runoff from a tributary area, referred to as a basin. The basin boundaries for each ravine were defined as the area tributary to the ravine drainage systems where the ravine drainage systems discharge to the Sammamish River. The three basins are contiguous and lie in a north-south direction, sloping north to the Sammamish River. The basin boundaries are illustrated on Figure 2 and Figure 4 (fold-out) at the end of this report. It is suggested the reader fold out Figure 4 while reading this report. The basin acreages, percent impervious, and overall runoff curve numbers are:

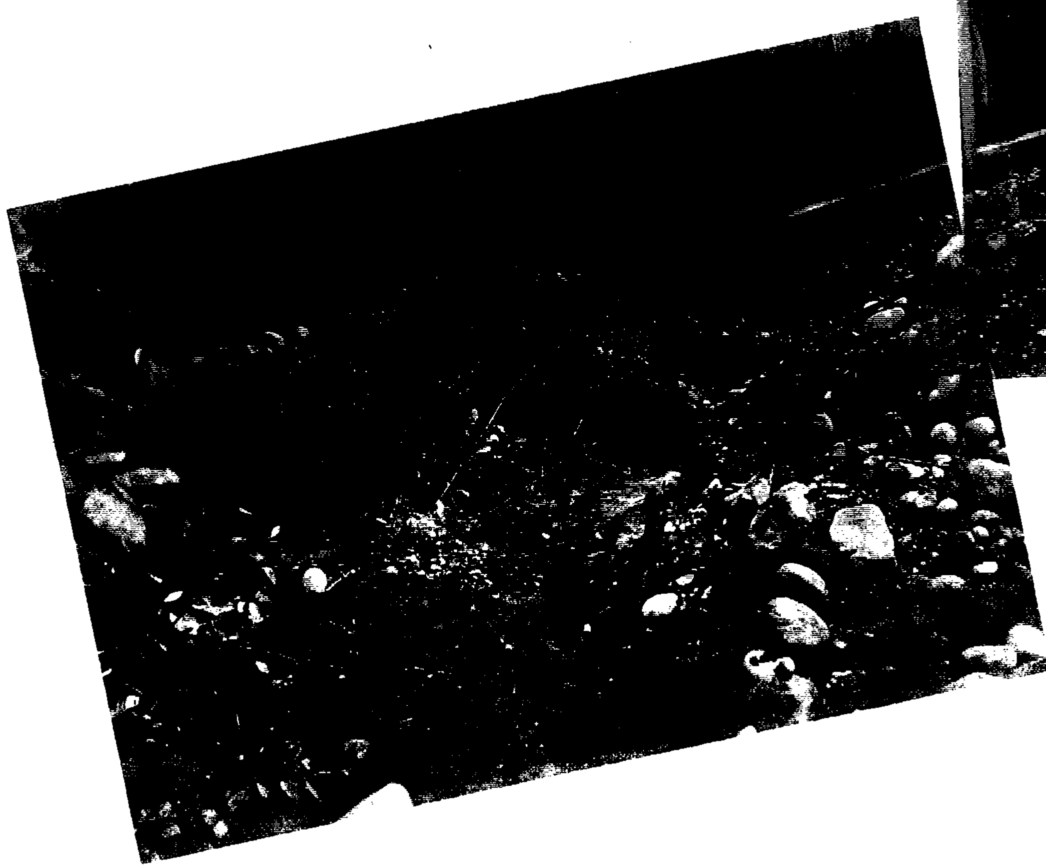
	Pervious Acres	Impervious Acres	Total Acres	Percent Impervious	Overall 1990 CN
Middle	102	22	124	18	82.3
West	27	6	33	18	84.3
East	35	2	37	5	81.7



SCHEMATIC PROFILE

PROBLEMS

1. LARGE VOLUMES OF SEDIMENT AND DEBRIS CAUSES PLUGGING AT INLET
2. INADEQUATE INLET CAPACITY EVEN WITHOUT CONSIDERATION OF SEDIMENT LOAD (Qinlet cap = 22 cfs, Q100 = 30 cfs)
3. SYSTEM IS PRIVATELY MAINTAINED



STATION 0+00
CULVERT ENTRANCE AT RAVINE BOTTOM
(LOT 4 QUINAULT ESTATES - DIVISION 3, DAY RESIDENCE)
FLOOD AND DEBRIS DAMAGE FROM JULY 13 EVENT

NOTE: LARGER ROCKS AND BOULDERS DEPOSITED IN THIS AREA. FINER MATERIAL DEPOSITED IN THE DOWNSTREAM AREAS.

QUINAULT ESTATES

PHOTOGRAPH 1

MIDDLE RAVINE – STATION O+OO
(AT CULVERT ENTRANCE)

R.W. BECK AND ASSOCIATES

LOT 10 QUINAULT ESTATES DIVISION 1 FOLLOWING JULY 13 EVENT

- IN ADDITION TO SEDIMENT DEPOSITION, SEVERAL FEET OF FLOODWATER ENTERED BASEMENT AND LOWER BUILDING LEVELS
- ONE OF APPROXIMATELY 20 RESIDENCES THAT WERE DAMAGED
- THE COUNTY HAULED AWAY 650 TO 700 CUBIC YARDS OF SEDIMENT FROM AREA RESIDENCES AND STREETS

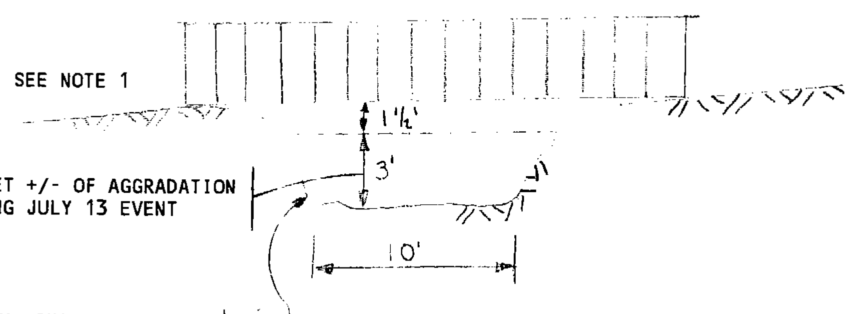
QUINAULT ESTATES

PHOTOGRAPH 2

MIDDLE RAVINE
JULY 13 EVENT DAMAGE

R.W. BECK
AND ASSOCIATES

STATION 1+20
(LOT 7 QUINAULT ESTATES DIVISION 3, READER RESIDENCE)



SCHEMATIC CHANNEL SECTION
(BEFORE AND AFTER JULY 13 EVENT)

QUINAULT ESTATES

PHOTOGRAPH 3

MIDDLE RAVINE
STATION 1+20
(FOLLOWING JULY 13 EVENT)

NOTE 1: REDUCED CHANNEL CROSS SECTION COULD RESULT IN NEW OVERBANK FLOWS AND FLOODING OF THIS RESIDENCE

R.W. BECK AND ASSOCIATES

Figure 4 also illustrates the existing drainage system and topography. The existing drainage system was based on County records and maps supplemented with a one-day field reconnaissance. Topography was based on 1953 aerial mapping.

Figures 2 and 4 also show a fourth basin referred to as the Lower Quinault Estates Basin. This basin does not contribute to any of the ravine drainage systems. It includes the northerly portion of Quinault Estates that is drained by the development's interior roadway ditch and pipe system, which eventually discharges to the Sammamish River between lots 14 and 15.

The ravine basins are described in the following paragraphs as having three components:

- Upper basin, which consists of the area tributary to the head of the natural steep ravines,
- Ravine, which consists of the steep valley walls,
- Lower basin, which consists of the alluvial fan where the Quinault Estate subdivision lies.

2. Drainage System

   a. Middle Ravine Basin. The upper portion of the basin consists of 67 acres and has two primary drainage pipe systems. The two pipe systems discharge flow at the top of the ravine, at the north side of NE 157th Street. One system includes a 24-inch-diameter system draining much of Inglemoor High School. The second system includes a 30-inch-diameter system draining a 2,000 foot section of Simonds Road NE as well as upland tributary flows from the south side of Simonds Road NE. These upland tributary flows enter the Simonds Road NE drainage system 300-feet south (upstream) of NE 157th Street through a 12-inch-diameter culvert.

   A small, approximately 60 foot by 200 foot wetland exists at the inlet to this culvert which provides some storage. This wetland was modeled as a HYD pond as discussed in the methodology section.

   The ravine (valley portion of the steep ravine) begins just north of NE 157th Street and extends in a northerly direction approximately 2,800 at a slope ranging from 10 to 20 percent. The ravine is fed by a combination of groundwater and surface runoff from the upper basin, creating a perennial stream downstream of NE 157th Street. The ravine geologic characteristics are discussed in detail in Section IV.

   At the lower portion of the basin (bottom of the ravine), the stream enters a 24-inch-diameter concrete pipe, located on the Day lot (see Figure 4 foldout and

Photograph 1). The 24-inch-diameter pipe system is in a drainage easement which extends north between lots 5 and 6 to NE 169th Street. At NE 169th Street, the pipe system is increased to 30-inch-diameter and extends northeasterly 550 feet along NE 169th Street where it turns north and goes 150 feet along a drainage easement to discharge into the Sammamish River. The lower 150-foot section includes three sediment control facilities, a 96-inch-diameter manhole, and two large sediment vaults (12-cubic-yard storage capacity each). The County installed the second sedimentation vault in November 1993. A profile of this system is included in Appendix B.

b. West Ravine Basin. The upper portion of the basin consists of 26 acres and predominantly consists of stormwater runoff from the 84th Avenue NE drainage system (Figure 4). This system includes a 12-inch-diameter pipe on the west side of the street and an open ditch with driveway culverts on the east side of the street. The east side ditch only conveys street runoff from half of 84th Avenue NE. The west side system collects 84th Avenue NE runoff as well as roadway runoff from 1,100 feet of Simonds Road NE and 20 acres of upland development. The development consists primarily of single family residential uses. Runoff from the upper portion of the basin is collected in pipe and ditch systems on both sides of 84th Avenue NE.

The ravine portion of the basin begins near the termination of 84th Avenue NE, 1,000 feet north of Simonds Road NE (Figure 4). The ravine extends north 830 feet to NE 169th Street. Although this report refers to the west ravine as a "ravine," a more accurate description would be a deeply incised drainage course on a very steep, erodible slope. The geologic conditions of the ravine is more thoroughly discussed under Section IV.

Ravine flows enter a 24-inch-diameter concrete drainage system located at NE 169th Street approximately 100 feet east of the extension of 84th Avenue NE. The pipe inlet includes an angled debris rack attached to the end of the pipe (see Photograph 6 in Section IV). From here the system goes west 100 feet then north 800 feet to the Sammamish River. As discussed under the problem history, ravine flows previously crossed NE 169th Street to enter the Quinault Estates drainage system; however, following a lawsuit, all flow from the ravine was diverted to the 24-inch-diameter system.

The west ravine also has some groundwater inflow. A profile of the west ravine is included in Appendix B.

c. East Ravine Basin. The upper portion of the east ravine basin can be divided into two areas; a west portion and an east portion. The west portion drains to the 88th Avenue NE drainage system and the east portion drains to a natural swale north of NE 163rd Street. The 88th Avenue NE system collects runoff from residential areas and a portion of the Inglemoor High School soccer field. This system discharges to an open channel north of NE 163rd Street. This discharge has eroded a drainage course referred to as the east ravine 88th Avenue Branch.

The east portion of the upper basin is smaller, and runoff from this area flows in a natural swale. The two short ravines join together to form the east ravine 450 feet north of NE 163rd Street. The two ravine branches are illustrated in Figure 5 in Section IV. The east ravine has some perennial groundwater flow.

Downstream of the confluence, ravine flows travel 800 feet north and enter a small sediment trap on the south side of NE 169th Street. The sediment trap provides around 20 cubic yards of storage and includes an 18-inch-diameter outlet with an overflow debris cage, and overflow ditches to the east and west. The 18-inch-diameter pipe extends north 60 feet and then is reduced to a 12-inch-diameter pipe which travels east 194 feet. The system then increases to 18-inch-diameter and extends 250 feet north through drainage easements to NE 170th Street.

The system again increases in size to 24-inch-diameter and goes northeast 300 feet where it discharges into an open channel which flows north to the Sammamish River. The open channel is in a drainage easement on the east side of Tract A of Quinault Estates. A profile of the east ravine is included in Appendix B.

3. Land Use

a. Middle Ravine Basin. The upper plateau of the middle ravine basin includes a combination of residential, public uses and open, undeveloped areas. Two schools, Inglemoor High School and Moorlands Elementary School, and one church contribute a significant portion (over 60 percent) of the basin's impervious area. Inglemoor High School is older and currently does not have a detention system. The drainage system drawings for Moorlands Elementary School were not available; however, it is also unlikely to have a detention system.

The ravine portion of the basin consists of lower density single (1DU/GA) family residential. Development consists of houses along the top of the ravine with the steep slopes remaining undeveloped.

b. West Ravine Basin. The west ravine basin land uses consist primarily of single family residential with some open undeveloped areas located in the north steeper portions of the basin. An exception is a church located along the south side of Simonds Road.

c. East Ravine Basin. The east ravine land uses include residential, open undeveloped areas, and a portion of the Inglemoor High School soccer field. The east ravine is much less developed than the other two basins.

4. Soils and Geology

The unstable nature of the ravine geology and soils is a primary contributing factor to the problem. Consequently, this study included an investigation of the area soils and geology. In addition, the SWM Prosecuting Attorney's Office retained AGI to perform geologic investigations following the July 13, 1993 flood. Portions of the conclusions and findings are incorporated into this report. The methodology for the soils and geologic investigations is described in Section III. The findings of the investigation are described in Section IV.

5. Environment

All three ravines are characterized by steep, undeveloped, wooded areas with year-round streams. The thick woods of Douglas Fir, Red Alder, Western Hemlock, Vine Maple, and Western Red Cedar provide good habitat for small mammals and birds. Stream riparian habitat is degraded due to excessive velocities and extensive erosion and sedimentation, which limit the establishment of riparian vegetation. These stream conditions also preclude any suitable fish habitat. There were no reports of a fishery in any of the three ravines, and none were observed during the field reconnaissance.

Much of each ravine is designated in the King County Sensitive Areas Map Folio as having a high landslide potential and a severe erosion hazard (King County 1990). A copy of the sensitive area figures excerpted from the Sensitive Areas Map Folio is included in Appendix D.

The streams in the middle, west, and east ravines are shown in the Map folio as "unclassified." During the field reconnaissance on September 29 and 30, 1993, all three ravines contained measurable flow. The time of year and lack of rainfall prior to the field visit suggests that the observed flow is base flow. In the middle ravine, flow was observed up to the top of the ravine at NE 157th Street. Flow in the west ravine was observed discharging from the end of the 84th Avenue NE pipe system. Flow in the east ravine was observed up to the confluence of the natural branch and 88th Avenue NE branch.

SECTION III

METHODOLOGY

This section describes the methodology used for investigating and evaluating the drainage problems. For each ravine, the investigation focused on three areas; hydrology, hydraulics, and the geologic and stream physical processes associated with landslides, erosion, and sedimentation. The hydrologic and hydraulic analyses were based upon the 1990 King County Surface Water Design Manual (KCSWDM). Important assumptions made during the hydrologic and hydraulic analyses are stated in this section and the results are described in Section IV. Geologic investigations were conducted through field investigations by geologists and project team members. The methodology for the geologic investigation is described in this section and the results are presented in Section IV.

A. Hydrologic Methodology

Peak runoff rates and runoff volumes for each ravine basin were estimated using the SBUH method utilized in the SWM "HYD" computer program. The HYD model was used to predict the system's hydrologic response for the 2-, 10-, 25- and 100-year 24-hour storm events. Precipitation data from the Norway rain gage (approximately 3 miles southeast of the problem area) was also used to simulate runoff from the January 9, 1990 storm and the July 13, 1993 storm.

The precipitation depths and storm distribution used in the analysis are as follows:

2-year/24-hour:	1.65 inches	SCS Type 1a distribution
10-year/24-hour:	2.47 inches	SCS Type 1a distribution
25-year/24-hour:	2.86 inches	SCS Type 1a distribution
100-year/24-hour:	3.52 inches	SCS Type 1a distribution
January 9, 1990 storm:	2.37 inches	Recorded Distribution (9-year return period for 24-hour precipitation)
July 13, 1993 storm:	1.67 inches	Recorded Distribution (1-year return period for 24-hour precipitation, over 100-year return period for 1-hour precipitation)

Copies of the precipitation distributions for the historical storms are included in Appendix A. Return periods for these storms are based upon the total precipitation during a specified time period and not peak runoff rates. It is important to note that the July event was a thundershower, and its intensity varied dramatically from location to location. For example, the Norway rain gage, located 3 miles southeast of the problem area, recorded a 100-year 1-hour return frequency, while the Juanita rain gage, located approximately 3 miles southeast of the Norway rain gage, recorded only a 1-year 1-hour frequency. Although the Norway rain gage provides the best available information, it is located out of the study area and actual precipitation (and runoff) in the study basin could have varied significantly.

Schematics of the HYD models, subbasin maps, and other back-up information such as a summary of subbasin characteristics, runoff curve number computations, time of concentration development, and model summary tables are included in Appendix A for each ravine basin.

For the evaluation of the existing system, peak runoff rates and volumes were estimated by the HYD model using curve numbers from both the 1979 King County Drainage Manual and the 1990 KCSWDM. Times of concentration were calculated per the 1990 Manual and were used in both the 1979 and 1990 curve number versions of the model; it is therefore acknowledged that flows generated with the 1979 curve numbers were not determined in strict adherence to the methodology outlined in the 1979 Manual.

An adjustment was made in the percent impervious for the residential land use classifications shown in Table 3.5.2.B (SCS Western Washington Runoff Curve Numbers) of the 1990 Manual. The adjustment was made to reflect the nature of the study area which consists of older residential development. Roofs and patios from older residential areas generally discharge to pervious areas rather than directly to storm drain systems. These impervious areas (non-effective impervious area) produce lower volumes of runoff than effective impervious areas because more infiltration occurs and the Tc's from individual impervious areas, such as roofs, lag considerably from Tc's of impervious areas connected to conveyance systems. The net effect is to lower peak runoff rates.

To account for the lower runoff from non-effective areas, runoff CN's from residential areas were reduced appropriately. The reduction was done by factoring down the percent impervious ratios from that shown in Table 3.5.2.B as follows:

- The percent impervious for older low density residential areas (1 to 2 DU/GA) was factored down by 60 percent

- The percent impervious for older medium and high density residential areas (greater than 2 DU/GA) was factored down by 40 percent

This method of accounting for non-effective impervious areas is consistent with work by USGS and others. This method of accounting for non-effective impervious areas is considered to be valid. However, it is not in strict compliance with the 1990 Manual, which states that all impervious areas are assumed to be 100 percent effective. To ensure that the recommended alternatives met the 1990 Manual requirements, the HYD model for recommended alternatives were simulated using both the non-effective impervious area adjustment and with impervious areas being 100 percent effective. The impervious areas from schools and churches were counted as 100 percent effective.

Runoff CN's used with storm data were also adjusted for antecedent moisture conditions in accordance with the 1979 Manual. The January, 1990 event was simulated for wet conditions (AMC III), and the July 1993 event was simulated for average conditions (AMC II).

The 1990 curve numbers were used to determine the level of protection (LOP) afforded by the existing system under current standards. Because the 1990 curve numbers generate higher peak flows for a given basin, LOPs calculated with the 1990 curve numbers are generally lower than those calculated with the 1979 curve number for the same basin.

Flows generated using 1990 curve numbers were also used to size conveyance elements for proposed alternatives, given that an upgraded system would be required to meet current standards e.g., safe conveyance of 100-year flows for out-of-basin diversions.

No significant (greater than 0.5 acre-feet) constructed detention systems were identified within the basin. There is natural storage occurring in a wetland located on the south side of Simonds Road NE approximately 300 feet south of NE 157th Street which was included in the middle ravine basin HYD model. This wetland provides between 0.5 and 1 acre feet of storage.

It is important to note that the HYD computer program methodology for hydrograph routing does not simulate flood flow attenuation when a conveyance system is undersized (resulting in surcharged flow and associated flood storage) unless this system is specifically modeled as a pond. Nor does the program incorporate channel routing algorithms that tend to attenuate peak runoff rates. Consequently, the analysis tends to be somewhat conservative, predicting higher flow rates than actually exist. The general modeling approach was to use the methods outlined in the KCSWDM and review the reasonableness of the results. For this study, the results were found to be reasonable, and no modifications were performed.

B. Hydraulic Methodology

The SWM "BWPIPE" direct-step backwater programs was used to evaluate the capacities of the three pipe systems at the base of the ravines. Each system was modeled from its outlet to the Sammamish River to its upper termination. Pipe system elevations and data for the middle ravine were based on County "as-built plans". Pipe systems for the east and west ravine were surveyed as a part of this investigation. The survey results are included in Appendix B.

The BWPIPE program computes backwater profiles through the system for a range of flows, considering inlet, outlet, and system losses. The BWPIPE program assumes the pipe system is clean and functioning properly, and therefore, does not account for hydraulic losses that result from sediment accumulation.

A Sammamish River tailwater elevation of 18.5 feet (National Geodetic Vertical Datum of 1929) was used for all backwater simulations. This elevation corresponds to the FEMA base (100-year) flood (FEMA, 1989).

C. Geologic Investigations

As previously discussed in Section II, the geologic investigations included a field reconnaissance of each ravine. For each ravine, the location at which the ravine flows enter the downstream pipe system was assigned Station 0+00. The walks were conducted upstream using a hip chain and taking photographs approximately every 200 feet. In addition, notes regarding geologic features such as substrate, bank failures, side inflows were noted and are included in Appendix D. Volume III of this study contains photographs taken during the field investigation. The County was supplied with one copy of Volume III.

SECTION IV

FINDINGS

The study findings have been divided into those relating to hydrology and hydraulics and those relating to geology.

A. HYDROLOGY AND HYDRAULICS

Table 1 summarizes the results of the hydrologic and hydraulic modeling. This table shows the simulated peak inflow at the base of each ravine, where the ravine flows enter the downstream pipe system. The table also shows the downstream hydraulic capacity, assuming that the pipes are flowing freely with no sedimentation or debris blockage.

1. Capacity Problems

Under the assumption of no sedimentation or debris blockages, there were three simulated capacity problems for the middle and east ravines, described below. There were no simulated capacity problems for the west ravine.

a. Middle Ravine. Inlet capacity of the middle ravine pipe system. Photograph 1 shows the pipe system inlet for the middle ravine. There is very little available headwater depth before overtopping occurs. The estimated inlet capacity of this pipe system is 22 to 25 cfs, whereas the simulated 25-year and 100-year peak flows were 21 and 30 cfs, respectively. Therefore, the inlet capacity has a level of protection (LOP) of approximately a 25-year event (1990 CN's), assuming there are no sediment or debris blockages. Realistically, given the high sediment and debris load in this system, the actual LOP is probably less than the 25-year event.

The pipe system downstream of the inlet, consisting of 24- and 30-inch-diameter concrete pipe, has a capacity of 35 cfs. Therefore, it has adequate capacity, assuming no sedimentation or debris blockage problems. A profile of this system is included in Appendix B.

b. East Ravine. The pipe system downstream of the east ravine consists of 12-inch-diameter, 18-inch-diameter, and 24-inch-diameter pipes. A profile of the pipe system is included in Appendix B. The bottom pipe section (pipe section number 1) is only a 2-foot long section between a catch basin and an open channel which flows north to the Sammamish River. This 2-foot-long section is located on the north side of NE 170th Street along the east edge of the Quinault Estates Tract A. Based on the backwater analysis, this last section of pipe would be submerged during the 100-year Sammamish River flood elevation of 18.5 feet, and its estimated conveyance capacity would be about 2 cfs. The section of pipe just upstream of the 2-foot-long section of pipe has a capacity of 14 cfs, which is well above the simulated 100-year peak rate of 8.9 cfs. Therefore, during the

Quinault Estates Study

Table 1

HYDROLOGIC AND HYDRAULIC ANALYSES SUMMARY

Storm Event	24-Hour Rainfall (inches)	Middle Ravine Peak flow Node M (1) (cfs) (90 CN)	Middle Ravine Peak flow Node M (1) (cfs) (79 CN)	Hydraulic Capacity of Downstream System (Assuming free of debris)	West Ravine Peak flow Node E (2) (cfs) (90 CN)	West Ravine Peak flow Node E (2) (cfs) (79 CN)	Hydraulic Capacity of Downstream System (Assuming free of debris)	East Ravine Peak flow Node D (3) (cfs) (90 CN)	East Ravine Peak flow Node D (3) (cfs) (79 CN)	Hydraulic Capacity of Downstream System (Assuming free of debris)
2-Year	1.65	7.9	6.6	The pipe system capacity is limited by the pipe system inlet, which has a capacity of 22 to 25 cfs. The downstream pipe system has a capacity in excess of 35 cfs.	3.6	3.3	Pipe system capacity: 30 cfs	1.4	0.7	Pipe section 8, shown on Figure 4, has a capacity of 8 cfs. The remainder of system has a capacity in excess of 14 cfs. Also, see note 6.
10-Year	2.47	15.9	131		7.6	7.3		4.2	2.7	
25-Year	2.86	20.4	17.1		9.7	9.4		5.8	5.5	
100-Year	3.53	28.8 (5)	24.7		13.6	13.3		8.9	6.6	
Jan. 9, 1990 (4)	2.37	27.4	23.5		10.1	9.9		7.9	6.9	
July 13, 1993 (4)	1.67	36.9	31.1		19.7	18.6		10.7	7.2	

Notes
(1) Middle ravine Node M corresponds to pipe system entrance on Day property
(2) West ravine Node E correponds to pipe system entrance at NE 169th Street
(3) East ravine Node D corresponds to pipe system entrance at NE 169th Street
(4) Historical precipitation taken from Norway rain gage.
(5) The 100-year estimate pre-developed basin flow for the middle ravine node M was estimated at 9 to 15 cfs, based on 1979 and 1990 CNs respectively. These pre-developed flow estimates are probably somewhat high because they were simulated with current condition time of concentrations.
(6) The capacity of the last section of pipe is limited during the 100-year Sammamish River flood elevations. Refer to text.

100-year Sammamish River flood elevations, any stream flows in excess of 2 cfs would spill out of the catch basin. This is not a problem, however, because any overflows would immediately enter the downstream open channel and flow directly to the Sammamish River. Therefore, this section of pipe is not considered a problem.

Pipe section number 8 is the second pipe section downstream of the bird cage inlet at the bottom of the ravine, (shown in plan on Figure 4, and profile in Appendix B) and consists of 12-inch-diameter concrete pipe. Its capacity is calculated to be 8 cfs. The 25-year and 100-year (1990 CN's) peak flow was simulated to be 5.8 and 8.9 cfs. Therefore, the upper catch basin of this section of pipe would overflow during the 100-year flow. The capacity of this pipe section could be increased to the 100-year LOP by raising the elevation of the catch basin grate and providing a locking lid. This catch basin was installed well below road grade and could easily be raised.

With the exception of pipe sections 1 and 8, described above, all other pipe sections have capacities of at least 14 cfs assuming no sediment or debris blockage problems.

One point of interest is that the County installed a system of overflow ditches starting near the bird cage inlet of the bottom of the ravine. These overflow ditches flow east and west along the south side of NE 169th Street. The elevation of the catch basin rim at the upstream section of pipe number 8 is 55.9. The elevation of the bird cage rim (upper catch basin of pipe section number 9) is 58.9. Although the elevations of the overflow paths were not surveyed, we believe that because of the significant elevation difference between the pipe number 8 upper catch basin rim and the bird cage rim, that flows would exit the pipe number 8 upper catch basin before flows would be diverted to the overflow ditches. Raising the catch basin grate on pipe number 8, as described above, would solve this problem.

c. Model Validation. No attempts were made to calibrate the hydrologic models. Further, validating the models by comparing simulated flows with historical observations is complicated by the fact that the flooding problems are dominated by sediment deposition and debris plugging problems rather than exceedance of hydraulic capacity.

Two historical storms were simulated for each ravine, January 9, 1990 and July 13, 1993. These historical events were simulated with both 1990 and 1979 curve numbers and the results are given on Table 3.

Historical observations for the 1990 event are limited. A flooding complaint was received by the owners of Lots 8 and 9 near the pipe entrance at the bottom of the east ravine. County maintenance crews visited the site following the complaint and found that much of the east ravine pipe system had been filled with sediment. We are aware of no major flooding problems on the middle or west ravines during this event.

Substantial data is available on the July 13, 1993 event. However, this event was also dominated by sediment transport and debris accumulation problems that led to flooding. Therefore, estimating actual observed flows is very difficult.

For this event, the pipe system entrances for all three ravines were overtopped, and all three ravines had debris accumulation problems, with the middle ravine being the most severe.

An estimate of the peak flows in the middle ravine for this event was made by AGI as a part of their geologic investigations. The flows were estimated by the average channel dimensions and the size of particles transported by the flood. AGI estimated the flow in the ravine of up to 180 cfs. Their methodology is described in a memorandum included in Appendix D. The estimate of 180 cfs is much greater than the peak flow simulated by the HYD model of 38 cfs. We believe the estimate of 180 cfs is high, but the 38 cfs may be low.

It is likely that the actual flows were greater than the 38 cfs simulated by the HYD model. The discrepancy between actual and simulated flows is most likely due to a greater precipitation in the Quinault Estates area versus what was recorded at the Norway rain gage, located three miles to the southeast. It was noted in Section III that the rainfall intensity of the July event varied significantly from one location to another. The Norway rain gage received 1.43 inches in one hour, while the Juanita rain gage, located three miles southeast of the Norway rain gage (the same distance between Quinault Estates and the Norway rain gage), received only 0.71 inches. It is possible that precipitation at Quinault Estates exceeded 1.43 inches per hour, and that flows were likewise greater than 38 cfs.

We believe that the actual flow was less than AGI's estimate of 180 cfs. At station 27+20 of the middle ravine, the stream must pass under a loose tree stump which has been wedged between the two stream banks. The stump essentially encloses the stream in a small open area similar to that of a very short section of culvert. Based on field observations, it appeared as though the stump was already in this location prior to the July 13 event. It also appeared that flows did not back up high enough at the upstream side of the stump to flow over the top of the stump. A photograph of this location is included in Volume III. The estimated open area available to pass flow is around 4 square feet. Although not directly applicable to this type of flow situation, culvert inlet capacity curves can be used to provide an approximation of flow regimes, given that the stump was not overtopped. Using this approach, the maximum flow during the July 13 event could have been in the range of 40 to 70 cfs. In summary, the July 13 event was an extreme event exceeding the 100-year flow rates for the system.

B. Geology and Channel Processes

A representative of AGI, Robert Palmquist, accompanied Jack Bjork and Mike Giseburt of R. W. Beck and John Bethel of King County on a field reconnaissance of the middle and east ravines. A letter report describing the findings of this field investigation, prepared by AGI, is included in Appendix D. A summary of these findings, as well as the findings from a R. W. Beck reconnaissance of the west ravine, is provided in the following paragraphs.

1. General

The Quinault Estates development lies on the south side of the Sammamish River Valley. The valley consists of a flat, alluvial bottom and relatively steep valley walls. Runoff from small, side drainage basins has created ravines in the valley walls. Alluvial fans have formed at the base of some ravines. Geology and soils maps indicate that the valley has eroded into the "Transitional Beds" which are being weathered into Kitsap Silt Loam. The Transitional Beds are non-glacial and glacial deposits underlying Vashon-Age Advance Outwash and Till. They consist of massive to bedded clay, silt, and fine to very fine sand which were deposited in lakes and streams. They are very unstable -- "The beds appear firm in outcrop but because of high water content and jointing, they become unstable in steep slopes; they are involved in numerous landslides" (Minard, 1983).

In a representative profile, the Kitsap Silt Loam (15-30 percent slope phase) consists of a 24-inch surface layer of silt loam over a substratum of silty-clay loam. Permeability is moderate in the surface layer but very slow in the substratum. Runoff from this phase is rapid and the erosion hazard and slippage potential are severe. The Kitsap series generally forms in glacial lake deposits (Synder, et al., 1973).

The upper plateau at the top of the ravines is underlain by Vashon Till and locally Vashon Advance Outwash. Vashon Till is "a nonsorted mixture of clay, silt, sand, pebbles, cobbles, and boulders in variable amounts. Though generally quite sandy, it locally contains much clay ..." (Minard, 1983). The underlying Advance Outwash "... typically is a ... clean, pebbly sand with an increasing gravel component higher in the section" (Minard, 1983).

The study area is designated in the King County Sensitive Area Map Folio as having a high landslide potential and a severe erosion hazard (King County, 1990).

2. Middle Ravine

a. General Geology. The general characteristics of the area geology were discussed above. Station locations described herein are relative to the pipe inlet in Quinault Estates, with the pipe inlet being station 0+00 and NE 157th Street being station 28+00 (Figure 5 and Photograph 4). A majority of the middle ravine lies within the Transitional Beds. From the Quinault Estates subdivision up to station 2+00 is an alluvial fan. Between station 2+00 and 26+70 the stream is within the Transitional Beds. Between stations 26+70 and 27+60, the in-place material becomes much sandier, suggesting a lens or pocket within the Transitional Beds or Advance Outwash. A 6-foot headcut is visible at station 27+60; upstream of that point, the creek has incised a small channel in the Vashon Till. A profile of the stream is included in Appendix B.

b. Drainage Basin. Currently the stream at the pipe inlet in Quinault Estates (station 0+00) drains an area of 125 acres. Step-by-step residential development and road-building in the upper basin has modified the original drainage patterns. It

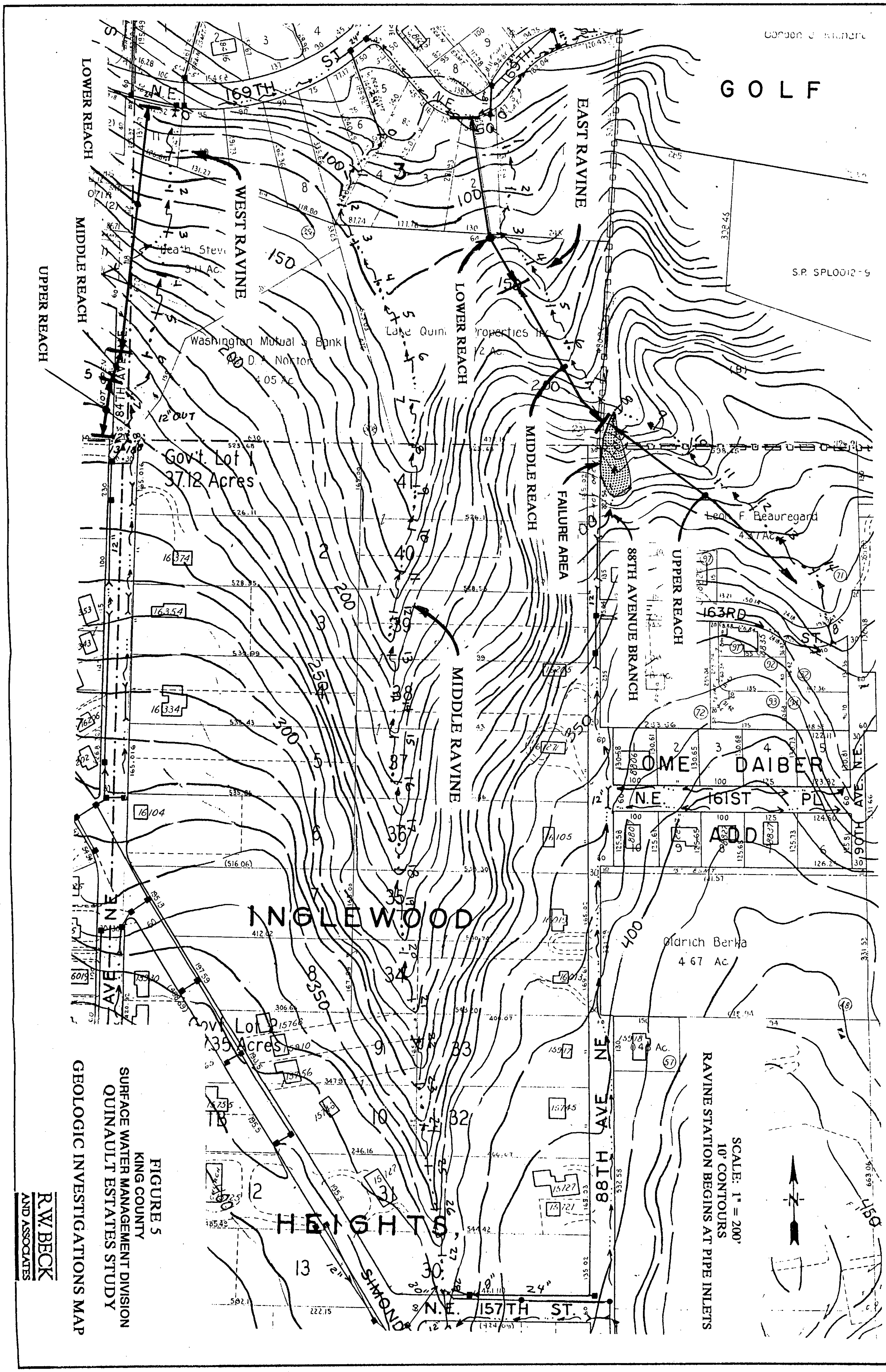
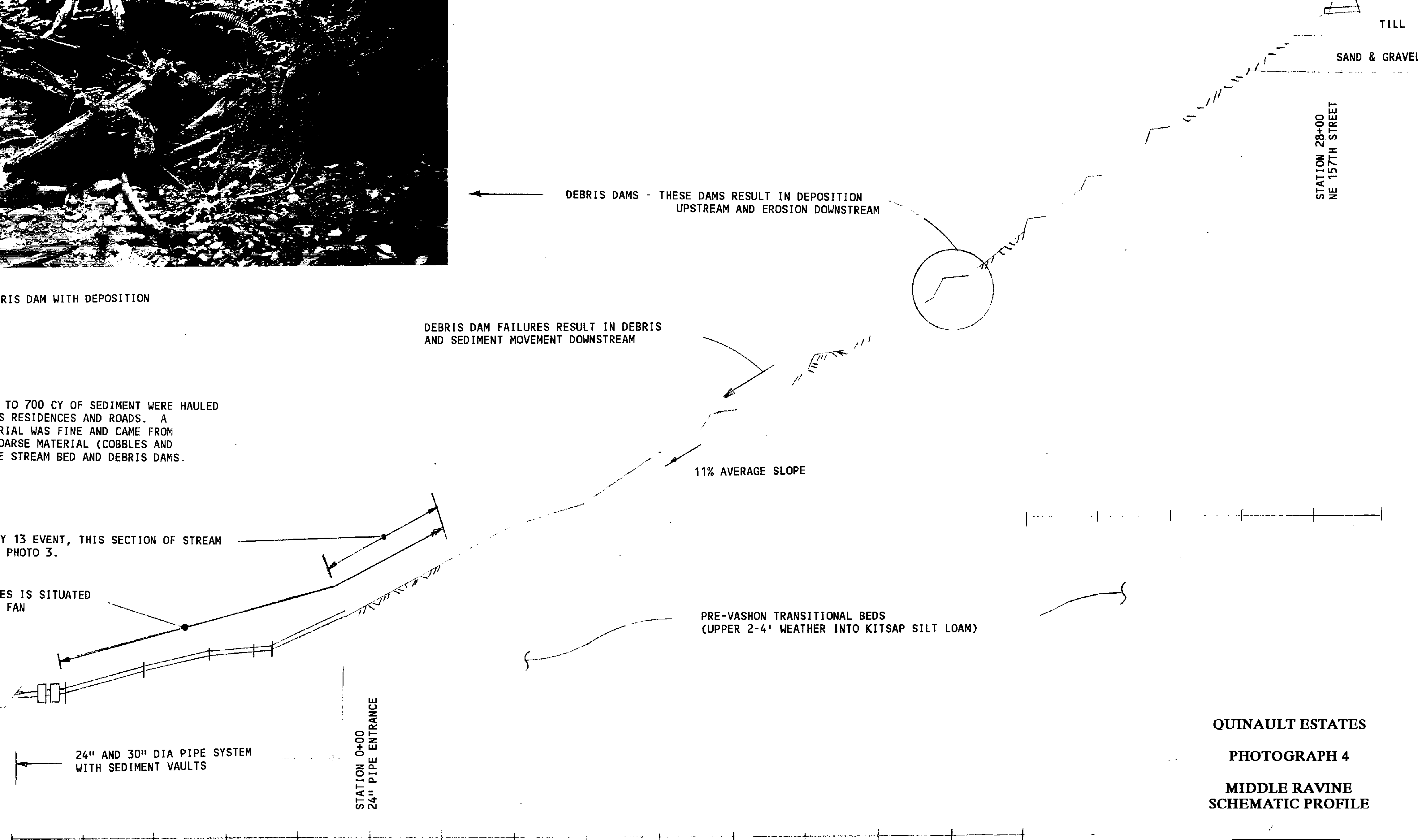


FIGURE 5
KING COUNTY
SURFACE WATER MANAGEMENT DIVISION
QUINAULT ESTATES STUDY
GEOLOGIC INVESTIGATIONS MAP

STATION 15+30 - 12 FT HIGH DEBRIS DAM WITH DEPOSITION
UPSTREAM AND SCOUR DOWNSTREAM



SCHEMATIC PROFILE

APPROXIMATE SCALE H: 1" = 300'
V: 1" = 60'

QUINAULT ESTATES

PHOTOGRAPH 4

MIDDLE RAVINE
SCHEMATIC PROFILE

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appears that the contributing area to the middle ravine has been reduced in recent years. Based on the 1953 10-foot contour map, the natural drainage basin was about 138 acres. This represents a decrease of 10 percent. While this would reduce ravine runoff, the current flows in the ravine are greater than natural conditions due to the effects of urbanization.

c. Ravine. The information provided here and in subsequent paragraphs uses significant portions of the draft letter report by AGI, dated November 12, 1993.

The ravine descends steeply to an alluvial fan on the floodplain of the Sammamish River. The middle ravine descends over a distance of approximately 2,800 feet from an elevation of 370 feet near NE 157th St. to 50 feet at its mouth in Quinault Estates with a mean longitudinal gradient of 11 percent. Its valley is V-shaped with steep hillslopes that rise directly from the channel with mean gradients between 27 percent (west side) and 42 percent (east side).

d. Valley-Forming Processes. The middle ravine is a young, actively-forming valley. Streams are actively down-cutting and extending headward, thus creating longer and steeper hillslopes. The result is a V-shaped valley with hillslopes rising directly from the stream channel. There are generally two processes occurring in this system, stream erosion and hillslope undermining and landsliding.

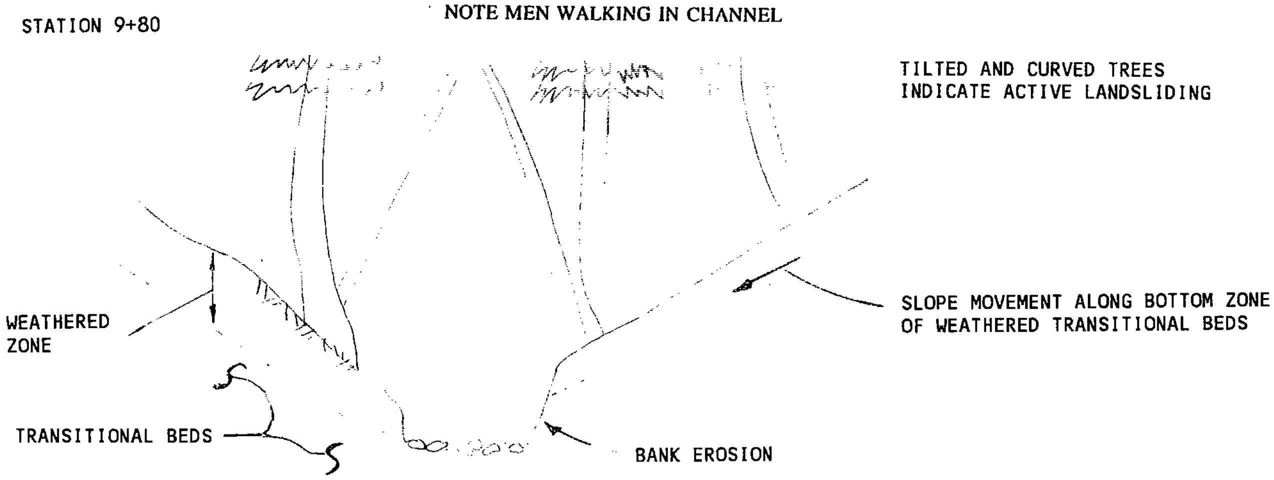
In a V-shaped valley, stream and hillslope erosion are interrelated. As the stream erodes downward, the hillslopes are undermined which cause them to fail. Then landslides transport soil from the hillslope into the channel. Stream erosion removes the toes of these landslides which continues to undermine the unstable hillslopes. The landslides extend headward until the entire slope is moving downward into the channel. In this state, the stream serves as a conveyor of soil from the hillslopes to the alluvial fan at its base.

The hillslope channel system is in balance. If the rate of stream erosion is increased, more sediment can be removed from the valley and the rate of hillslope erosion increases and the valley becomes deeper and wider. If the rate of stream erosion decreases, sediment from the hillslopes accumulates in the channel, and hillslope erosion decreases. The rates of stream and hillslope erosion are controlled by the quantity of runoff.

The hillslopes in the project area are naturally unstable because of the underlying silts. This instability is promoted by infiltration and groundwater seeping into overlying soils. In this respect, landsliding in this area is independent of the rate of channel erosion. Photograph 5 illustrates the unstable nature of the hillslopes.

Infiltration occurs during all storms. Thus, landsliding may continue to deliver sediment to the channel during years without intense storms. During these years, channel erosion may not be capable of removing the sediment, and it will accumulate. The sediment will be stored in the channel until a large, intense

NOTE MEN WALKING IN CHANNEL



SCHEMATIC SECTION

QUINAULT ESTATES

PHOTOGRAPH 5

MIDDLE RAVINE
SCHEMATIC SECTION

1. SMALL DEBRIS DAM WITH DEPOSITION UPSTREAM AND SCOUR DOWNSTREAM.
2. BOULDERS ON TOP OF LOGS INDICATING MOVEMENT OVER LOG DURING JULY 13 EVENT.

R.W. BECK
AND ASSOCIATES

storm flushes it downstream. Thus, sediment removal from the channel is sporadic and episodic.

The July 13 event was one of these normal, but infrequent, episodic flushing events as indicated by an analysis of sediment sources and probable flood recurrence interval.

e. <u>Hillslope Characteristics</u>. Field inspection indicates the east and west hillslopes are unstable with numerous shallow debris flows. These are composed of brown silts and clays. Their surface is covered by underbrush and tilted and bent trees up to 3 feet in diameter. The large deformed trees indicate long-term continued movement. Photograph 5 shows a typical cross-section through the ravine. The topographic contour lines on the east hillslope have two large concavities that are interpreted to be large, composite landslide scars. Given the steep slopes and the presence of the Transitional Beds, landslides are to be expected.

f. <u>Channel Characteristics</u>. The stream channel geometry in the middle ravine varies considerably. Where a floodplain is present, the channel is trapezoidal in cross-sectional shape and approximately 6- to 8-feet wide and 1.5- to 2-feet deep. Banks are typically 1-1/2- to 4-feet high. In reaches without a floodplain, the channel may be narrower (2 to 3 feet) and deeper because of landslides and outcrops of the Transitional Beds. In other reaches, the channel is wider (up to 12 feet) and shallower. Where aggraded, as in the lower 200 feet and upstream from a debris dam, it may be from 6- to 15-feet wide.

The channel character changes along its length. The lower 200 feet was aggraded with sandy gravel about 3 feet during the July 13 event to the level of the floodplain (see Photograph 3). This lower 200 feet probably represents the upper end of the alluvial fan.

Between stations 2+80 and 22+80, the channel contains 18 debris dams. The debris dams consist predominantly of tree trunks and limbs. The dams range from 2 to 12 feet high with a mean height of about 3 feet. Photograph 4 shows the largest debris dam observed. The lower dams have been breached with partial removal of upstream sediment. On some of the dams, individual cobbles lay on top of the upper logs. Photograph 5 illustrates a boulder on top of a small debris dam.

Typically, the channel bed downstream from a debris dam is either sandy gravel or scoured silt of the Transitional Beds. Above each debris dam, the channel is aggraded by sandy gravel. Aggraded reaches (above debris dams and within the lower 200 feet) are estimated to cover 20 percent of the total channel length or approximately 600 feet.

Many reaches have beds covered with sandy gravel, occasional boulders (up to 2 feet in diameters) and large logs. These reaches are interpreted to be more stable and are estimated to cover half of the total channel length or approximately 1,400 feet.

In scoured reaches, the gravelly bed has been removed and the silt of the Transitional Beds is exposed. These reaches are common upstream of station 1+50 and become predominant upstream of station 19+00.

The scoured reaches appear to be steeper. In one scoured reach, the gradient was 20 percent. In several reaches, narrow channels have been eroded into the silt. Scoured reaches are estimated to cover 30 percent of the total channel length, or about 800 feet.

g. Sediment. Approximately 1,200 cubic yards of sediment were transported during the July 13 event. The sediment was derived from erosion of channel banks and bed, as well as from small landslides entering the channel. The material was deposited within the lower reaches of the channel, behind debris dams, and within Quinault Estates. Photographs 1 and 2 illustrate sediment deposition within Quinault Estates. Sediment quantities were estimated from visual inspection of the channel (combined with simplifying assumptions) and truck tallies of sediment material hauled away following this event (see Table 2). The quantities should be considered order-of-magnitude estimates. They are included in this report to assess the relative importance of various processes and sediment sources.

Extensive bank erosion occurred during the July 13 event. Small debris flows entered the channel upstream of station 2+20. In several reaches with floodplains, the roots of small trees and brushes protruded from the top of the 2-foot-high bank. They appeared to be fresh and are interpreted to have been exposed by flood scour. If so, they indicate bank erosion of as much as one foot by water at a depth of 2 feet.

The quantity of materials eroded from channel banks during the July 13 flood is estimated to be 700 to 800 cubic yards. Of this amount, 100 to 200 cubic yards of fine-grained sediment were generated by the observed bank erosion and 500 to 600 cubic yards were produced by debris flows. Perhaps an additional 50 cubic yards were derived from less obvious bank scour. About 90 percent of the material was eroded between stations 4+00 and 14+00.

Extensive erosion of the channel bed also occurred. The bed consists of gravelly reaches, alternating with scoured reaches exposing the Transitional Beds. The amount of bed erosion can be inferred from the quantity of coarse-grained sediments deposited behind debris dams, in the aggraded lower channel reach and in Quinault Estates.

Deposition of 350 cubic yards of sandy gravel occurred within the channel. Approximately 200 cubic yards were deposited in the lower 200 feet of channel and 150 cubic yards were trapped by debris dams. Combining these estimates with the 120 cubic yards of gravel deposited in Quinault Estates yields 400 to 500 cubic yards of bed erosion.

Table 2

Order of Magnitude Sediment Balance of Middle Ravine During July 13 Event

	Cubic Yards
Sediment Sources	
Bed erosion	200
Erosion of Banks and Failures	750
Released from Debris Dams	250
Upper Drainage Basin	Minimal
Total	1,200
Sediment Depostion	
In Quinault Estates Roads and Residences	750
Lower 200 feet of Channel	200
Trapped in Debris Dams	150
In Sammamish River	100
Total	1,200

This analysis indicates the source of the sediments deposited in Quinault Estates were the banks and landslides within the middle ravine. Most of the coarser sediment was retained within the ravine behind debris dams and channel aggradation induced by the debris-blocked culvert and reduced gradient.

h. Conclusions. The July 13 event was a flood produced by an intense rain storm. The recurrence interval of this storm was greater than 100 years. Quinault Estates is located on an alluvial fan at the mouth of the east and middle ravines. The alluvial fan was formed by repeated episodes of flooding and sediment deposition during historic and prehistoric time.

The sediments were derived from erosion of the ravine. Flood waters eroded channel and landslide deposits. The episodic removal of these sediments from the ravine is part of the normal geologic processes which formed it. Continued erosion, sediment transport and deposition downstream of the middle ravine activity should be anticipated.

3. West Ravine

a. Geology. The geologic setting or stratigraphy of the west ravine is similar to the middle ravine. Vashon Till overlies the other units and exists as a plateau in upper part of the basin. The bottom of the Vashon Till unit in the west ravine basin is encountered in the roadside ditch along 84th Avenue NE. Consequently, it is above the ravine.

Underlying the Till are the Transitional Beds. The ravine is within this unit. The Transitional Beds were observed throughout the incised ravine and consisted primarily of compact, clayey silt with a weathered soil horizon of several feet. Minard (1983) mapped much of the west ravine area as a landslide deposit. Observations along the incised ravine did not reveal large quantities or thicknesses of landslide material. In general, the banks and hillslopes in the west ravine area appeared to be more stable than those in the middle ravine.

b. Findings.

(1) Drainage Area

The contributing drainage area of the west ravine has expanded in recent years. Prior to any development in the basin (under natural conditions), the area draining to the culvert under NE 169th Street was about 8 acres. Urbanization and original construction of NE Simonds Road and NE 84th Avenue added 3 acres. In 1977, NE Simonds Road was widened and the drainage system was modified and collected runoff from the widened sections of the road and from a large area south of NE Simonds Road. This modification added 22 acres, creating a total drainage area of 33 acres. Increasing the drainage area to the ravine by 310 percent has increased the peaks and quantities of flood flows, and the rate of erosion.

(2) Ravine and Hillslope

The ravine can be divided into three reaches: the upper reach, the middle reach and the lower reach (Figure 5 and ravine profile in Appendix B.) The upper reach lies 200 linear feet below the pipe outfall at the north end of NE 84th Avenue. The middle reach is a 80-linear-foot section between the upper and lower reaches. The lower reach extends from the middle reach 550 linear feet down to NE 169th Street. The hillslope along the path of the flow has an overall slope of 21 percent.

The upper reach is a scoured section below the pipe outfall. In this area, there was probably no natural channel prior to construction of the outfall. Flows have formed a channel by erosion. The channel is 5 feet deep at the outfall and gets shallower in the downstream direction until the channel is undefined and has no banks. The total volume of channel erosion that has occurred over time in the upper reach is 50 to 60 cubic yards. The channel slope in the upper reach is about 10 percent.

Runoff in the upper reach also includes flows from the roadside ditch on the east side of 84th Avenue NE. This ditch only carries runoff from the street itself. At the north end of 84th Avenue NE, however, the ditch does not flow into the channel previously described, but flows down the hill overland. The ditch flow is ultimately collected by the channel about 400 linear feet downhill.

The downstream end of the upper reach is the beginning of the middle reach. The middle reach is only 80 feet long. In this reach there is no defined channel, and deposition rather than erosion is taking place.

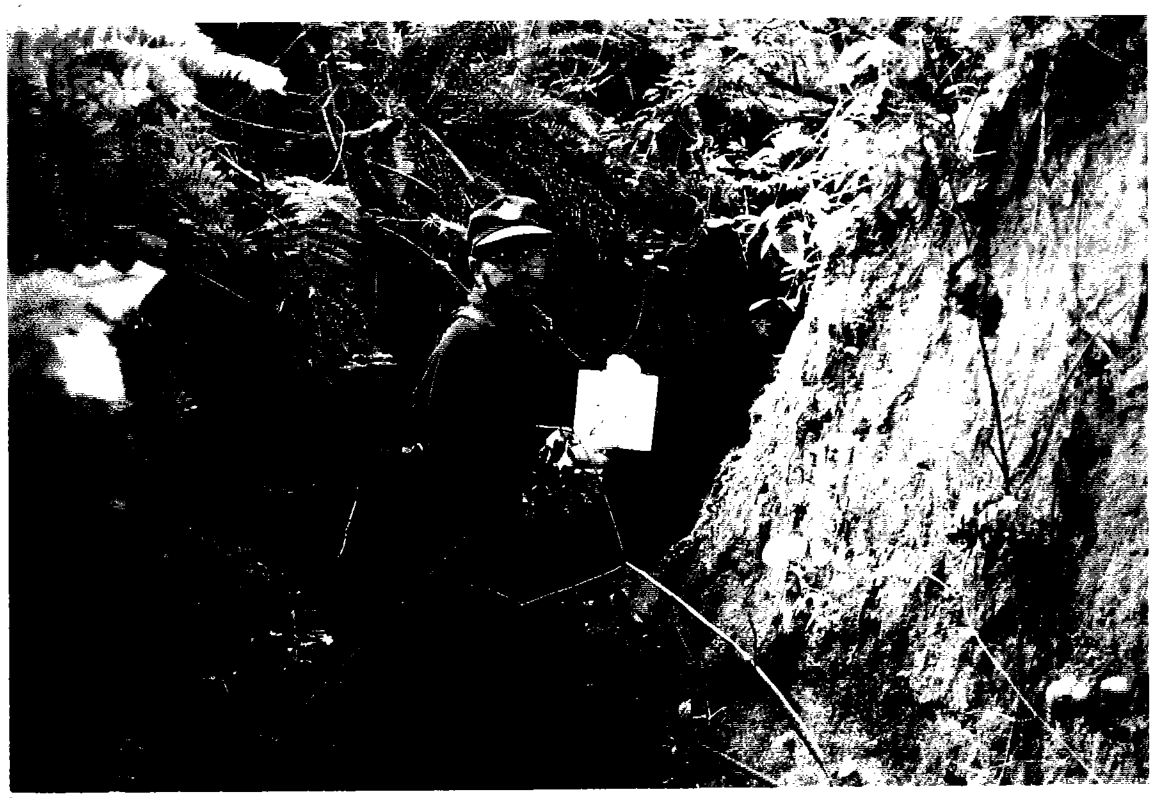
The most active section of the ravine is the lower reach. This section is an incised channel superimposed on the older, much smaller natural swale. The eroded channel is probably the result of dramatic increases in runoff flowing down the swale. The overall slope of this reach is about 26 percent with occasional vertical drops of up to 3 feet. Steeper sections have gradients over 30 percent. Photograph 6 illustrates these features.

Three processes are active in this reach: downcutting, bank erosion, and headcutting. Downcutting or bed erosion has created an incision averaging 5 feet deep. This erosion is through the relatively erodible, weathered soil and well into the compact silt of the Transitional Beds. Occasional cobbles and boulders in the Transitional Beds have remained, creating some armoring. However, the very steep channel and high velocities can transport the larger material. Erosion will continue.

The second related process is bank erosion. The channel banks and the hillslopes are more stable than the middle ravine. The channel banks have an average sideslope of 3 horizontal to 1 vertical. However, the channel will widen in time as part of gully formation. The third process is

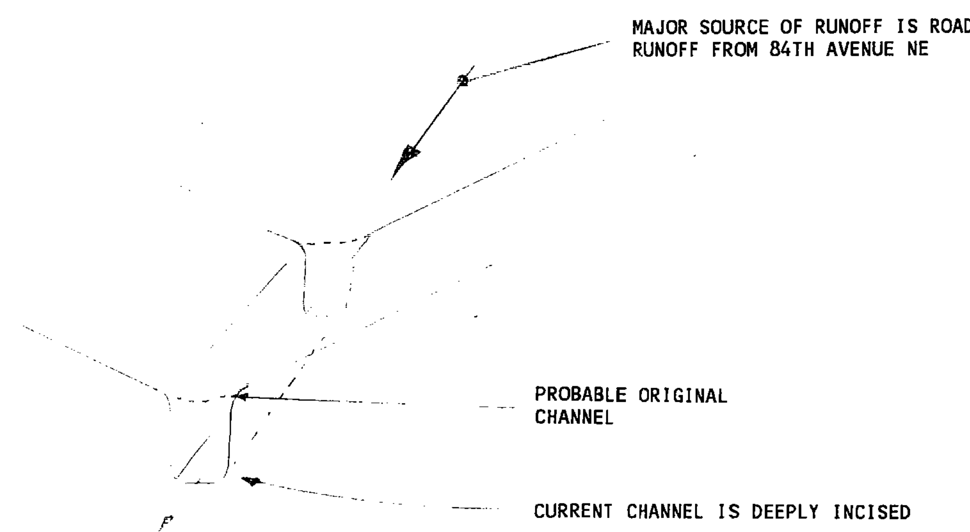
STATION 0+00
PIPE INLET AT BOTTOM OF RAVINE. DEPOSITION OF BED AND BANK MATERIAL AND PLUGGING BY WOODY DEBRIS GREATLY REDUCES INLET CAPACITY AND RESULTS IN OVERTOPPING OF NE 169TH STREET. NOTE LARGER MATERIAL (COBBLES AND BOULDERS WERE DEPOSITED HERE AND FINER MATERIAL CONTINUED DOWNSTREAM.

STATION 0+00 - LOOKING SOUTH ACROSS NE 169TH STREET



STATION 1+25

THE CHANNEL IS DEEPLY INCISED THROUGH TRANSITIONAL BEDS. THIS INCISION IS OCCURRING ALONG 680 OF THE 830 LF OF CHANNEL, WITH AN AVERAGE DEPTH OF 5 FT. ORIGINALLY, THIS PROBABLY WAS A VERY SMALL CHANNEL DRAINING A SMALL AREA. WITH AN INCREASE IN CONTRIBUTING SURFACE RUNOFF, CONTINUED SCOUR IS EXPECTED.



SCHEMATIC SECTION

QUINAULT ESTATES

PHOTOGRAPH 6

WEST RAVINE

R.W. BECK AND ASSOCIATES

headcutting. This describes the upstream movement of the beginning (or head) of the ravine.

At the downstream end of the lower reach is a short, 20-linear-foot section where the channel meets the pipe inlet (Photograph 6). The relatively large quantities of woody debris and fine and coarse material have occasionally plugged the pipe inlet. There is virtually no sediment storage volume at the pipe inlet, and even a small sediment volume, e.g. 5 cubic yards, could plug this inlet causing overtopping of the road. It was estimated that in the past, the deposition of about 5 cubic yards have occurred in this section when the pipe has become plugged.

The ravine in the lower reach was created by the erosion of about 550 cubic yards of material, based on field estimates. Most of this material has probably eroded since 1977 when diverted flows were added upstream. When combined with the erosion in the upper reach (550 plus 50 cubic yards), the average erosion for the entire ravine is 35 to 40 cubic yards per year, although the actual rate of erosion is highly variable from year-to-year.

The west ravine can be characterized as a discontinuous gully, as described by Leopold, Wolman and Miller, (1964):

> A common characteristic of a new gully system is discontinuity along the length of the developing channel. A discontinuous gully system is characterized by a vertical headcut, a channel immediately below the headcut which is often slightly deeper than it is wide, and a decreasing depth of the channel downstream. Where the plane of the gully floor intersects the more steeply sloping plane of the original valley floor, the gully walls decrease to zero in height and a fan occurs.
>
> As the process continues with time, a stage will be reached when the headcut of the downstream gully meets the toe of the one upstream and the two discontinuous gullies coalesce.
>
> As the two gullies coalesce, the slope will increase, increasing the erosive capability of the stream. This description underscores the fact that the gullying of the west ravine is a relatively recent occurrence, in geologic terms.

The material in the west ravine has been removed in three ways (in order of decreasing importance):

- Conveyed through the pipe system to the Sammamish River (fine material)
- Deposited at the inlet and removed by county maintenance crews (coarser material)

- Transported with flood overflow across NE 169th Street onto residential lots and streets where it was removed by residents and county maintenance crews

If the pipe inlet remains open, fine-grained and sand-sized material could be transported through the pipe to the river. The quantity of material which has been transported through the pipe system to the Sammamish River or carried over NE 169th Street is unknown.

4. East Ravine

a. Geology. The geology of the east ravine is similar to the middle and west ravines. A Vashon Till cap overlies the other units and comprises the upper or south end of the basin. Under the Till are the Transitional Beds, as discussed previously. Exposed at the surface of the middle portion of the basin is the PreVashon Till, according to Minard (1983). His map shows it as a 200-foot-wide swath across the basin. The lower portion of the basin has been mapped as landslide deposits. A map showing the location of these units is in Appendix D.

b. Findings.

(1) Drainage Basin

As with the west ravine, the drainage area of the east ravine has been increased by construction of ditches and piped drainage systems. The natural drainage basin had an area of 26 acres. The construction of roadside ditches and culverts for 88th Avenue NE around the early 1960s added 10 acres to the upper basin. This addition represents a 38 percent increase in area. Proportional increases in runoff have resulted.

(2) Ravine

The existing ravine consists of a natural channel about 1,400 feet long and a 400-foot-long branch which has developed from the outfall of the roadside ditch from 88th Avenue NE (Figure 5). In this report, this will is the 88th Avenue Branch. A schematic profile and photographs of the ravine are shown in Photograph 7.

The natural channel can be divided into three reaches. The lower 450-foot reach has an overall slope of 17 percent. Currently, there are 3 debris dams, each 2 to 3 feet high. The average width of the active channel is about 4 feet. Both bed scour and deposition were observed so net aggradation or degradation is not evident in this reach. The lower 300 feet of this reach has experienced filling and grading along the west bank for installation of a sewer line, although this work has not significantly reduced the flow area. Both banks are relatively stable, however. Erosion of the banks by the stream is minor. At the bottom of the reach is the intake to the pipe system

(Photograph 7). Deposition occurs at this point with a maximum available volume of about 20 cubic yards.

The middle 350-foot reach has a slope of 21 percent. It currently has 2 debris dams, 3 and 5 feet high. This reach is clearly aggrading. The middle reach also has a channel that is twice as wide as the lower reach. This is consistent with aggradation. The middle reach also contains hillslopes which appear unstable. Trees of varying sizes are tilted or have curved trunks (Photograph 7). Four stream-side slope failures were observed in this reach. Bank erosion in this reach is greater than in the lower reach.

The upper 600-foot reach of the natural channel is above the confluence with the 88th Avenue Branch. The upper reach has an overall slope of 17 percent, but also contains two steep chutes scoured into the compact silt of the Transitional Beds with slopes over 35 percent. These two chutes total about 140 feet in length with a headcut at the upper end. The depth of incision varies from 1-1/2 to 7 feet. The active channel width is 3 to 4 feet. Upstream of the headcut at station 10+50, runoff flows across a pasture in a more gently sloping grassy swale. No erosion or slope stability problems were observed upstream of the headcut.

The most dramatic conditions exist along the 88th Avenue Branch (Photograph 7). From the pipe outfall, the channel has incised about 6 feet for a distance of 175 feet. This incision terminates at a large 3,200-cubic-foot bowl. The failure is a 175-foot-long elongated bowl with a maximum depth of roughly 25 feet and a maximum top width of about 60 feet. The failure was probably formed by several large slides, numerous small slides, frequent surface spalling, and erosion. Large cracks are visible around the top, indicating additional failures could occur. The initial failure occurred in the 1970s, according to court statements of a long-time resident.

Currently, there is a considerable quantity of material from failures resting at the bottom of the bowl that could be easily eroded and transported by runoff. During the field visit in late September, significant groundwater inflow was observed at the lower end of the bowl; this was probably a contributing factor in the initial failures. The original hillslope had a gradient of about 40 percent.

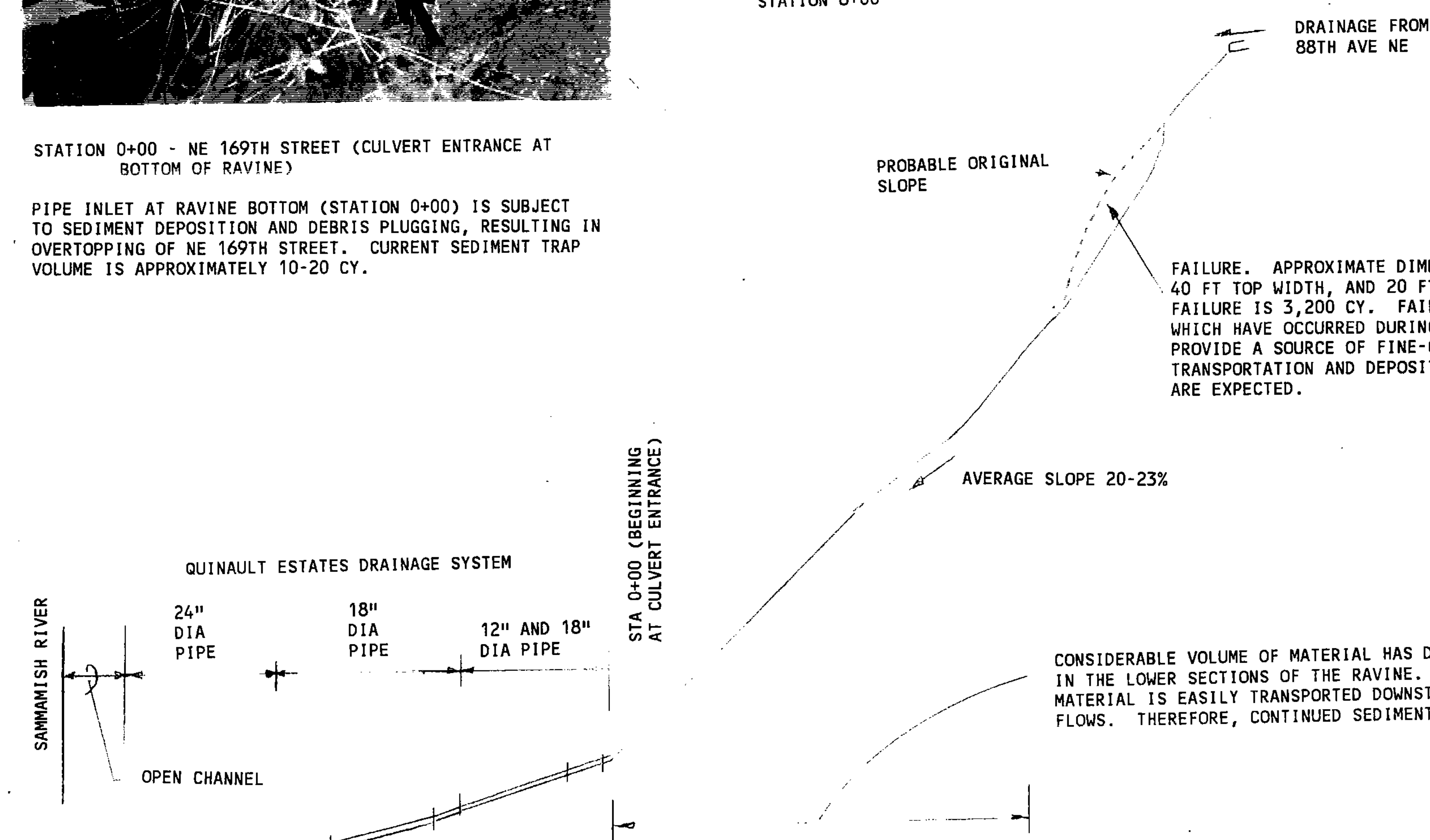
An attempt was made to roughly quantify the sediment balance in the ravine since the 1970s. The purpose of estimating the balance is to assist in understanding the processes that are ongoing in the east ravine and to better evaluate the proposed alternatives. Based on field estimates, about 3,400 cubic yards of material has been released into the channel (3,200 cubic yards from the failure and 200 cubic yards from upstream of the failure). Additional, less dramatic quantities of sediment would be expected to enter the stream from the remainder of the drainage basin.

STATION 0+00 - NE 169TH STREET (CULVERT ENTRANCE AT BOTTOM OF RAVINE)

PIPE INLET AT RAVINE BOTTOM (STATION 0+00) IS SUBJECT TO SEDIMENT DEPOSITION AND DEBRIS PLUGGING, RESULTING IN OVERTOPPING OF NE 169TH STREET. CURRENT SEDIMENT TRAP VOLUME IS APPROXIMATELY 10-20 CY.

STATION 6+00

STATION 10+25 - 88TH AVENUE NE BRANCH (UPPER PORTION OF FAILURE)



FAILURE. APPROXIMATE DIMENSIONS OF 175 FT LONG, 40 FT TOP WIDTH, AND 20 FT DEEP. ESTIMATED TOTAL VOLUME OF FAILURE IS 3,200 CY. FAILURE IS THE RESULT OF SEVERAL SLUMPS WHICH HAVE OCCURRED DURING THE PAST 20 YEARS. FAILURES PROVIDE A SOURCE OF FINE-GRAINED MATERIAL FOR DOWNSTREAM TRANSPORTATION AND DEPOSITION. CONTINUED SLOPE FAILURES ARE EXPECTED.

CONSIDERABLE VOLUME OF MATERIAL HAS DEPOSITED AND REMAINS IN THE LOWER SECTIONS OF THE RAVINE. THIS FINE GRAINED MATERIAL IS EASILY TRANSPORTED DOWNSTREAM DURING HIGH FLOWS. THEREFORE, CONTINUED SEDIMENT TRANSPORT IS EXPECTED.

SCHEMATIC PROFILE (EAST RAVINE/88TH AVENUE NE BRANCH)

APPROXIMATE SCALE H: 1" = 300'
V: 1" = 60'

QUINAULT ESTATES

PHOTOGRAPH 7

EAST RAVINE

R.W. BECK
AND ASSOCIATES

Our rough estimate of what happened to that material is as follows, in order of decreasing importance:

- Transported to the Sammamish River via the pipe system and deposited in the river (1,500 to 2,500 cubic yards).

- Deposited and currently stored in the ravine channel (600 to 1,000 cubic yards).

- Removed at the pipe intake by county maintenance crews after significant flows (100 to 300 cubic yards).

- Transported with flood overflows across NE 169th Street and deposited on streets, in ditches and on residential lots of Quinault Estates during significant flows (200 to 500 cubic yards).

This balance suggests that the two more obvious deposits of material (at the inlet and in Quinault Estates) are relatively small. The balance also suggests that much of the material has already been transported through Quinault Estates. This is consistent with the comments of the Department of Fisheries regarding the impact of deposition and river dredging on anadromous fish in the Sammamish River. The most important value in this estimate is the quantity of material that remains in the channel upstream of the pipe inlet. This material, along with material resting at the base of the failure on the 88th Avenue Branch, can still cause considerable damage and continued problems.

c. Alluvial Fan. Downstream of the ravine, an alluvial fan has formed from material transported by the east and middle ravines. The alluvial fan represents the area on which many of the homes of Quinault Estates are built. Moving north toward the Sammamish River, the alluvial material from the middle and east ravines gives way to the flatter alluvial plain of the Sammamish River. This plain is composed of fine-grained material. According to soils maps, this transition is made 200 to 300 feet south of the Sammamish River.

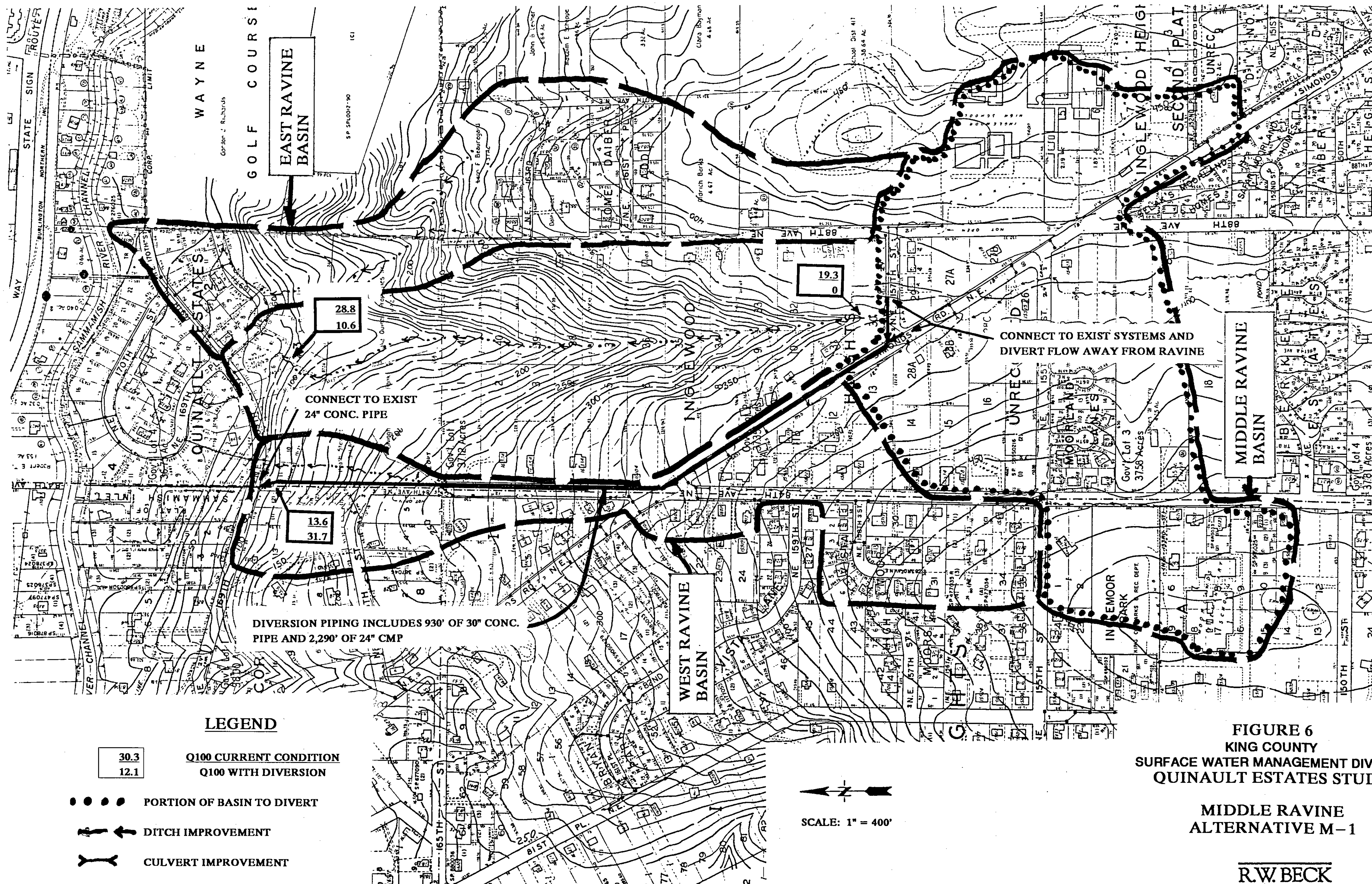


FIGURE 6
KING COUNTY
SURFACE WATER MANAGEMENT DIVISION
QUINAULT ESTATES STUDY

MIDDLE RAVINE
ALTERNATIVE M–1

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